Use of light-weight steel strapping as external reinforcing to concrete beams

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USE OF LIGHT-WEIGHT STEEL STRAPPING AS EXTERNAL REINFORCING TO CONCRETE BEAMS

BY

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THESIS

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ABSTRACT

As a result of some recent failures in reinforced concrete construction throughout the United States, a study was made by several national authorities to find some remedial measures of strengthening existing structures without rebuilding. A system was developed, using high strength steel bands applied externally to beams, to act as stirrups, which has been highly successful. It not only greatly increased the strength of beams already in place but was used to repair members which had already failed.

The application of these bands was made under very exact specifications, using closely controlled procedures, with many physical refinements, in order to obtain a high degree of efficiency. Furthermore, the materials and tools employed were of special design, not readily available on the open market, that had to be specially procured direct from the manufacturer.

This author wanted to know if the above special tools and equipment were not available, could ordinary strapping material of lesser strength be used and still get acceptable results.

This study indicates a strength of a satisfactory and acceptable degree may be achieved by using common materials applied in an ordinary manner without the refinements established by previous investigations.

ACKNOWLEDGMENT

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Acknowledgment is also made of the aid received from Mr. William E. Schaem, Office of the Chief of Engineers, U.S. Army, for supplying references and data which provided background material for this study.

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INTRODUCTION

Many years ago, before the use of reinforced concrete began to gain widespread favor in building construction, the design of concrete beams followed only basic principles of statics and mechanics. Many unknowns were either not thought of, overlooked, or ignored. If an early designer did consider some of the unknown factors that might have an effect on the finished product, he probably did one of two things:

1. Incorporate a large safety factor into the design to take care of the unknowns

or

2. Abandon concrete altogether in favor of more commonly used materials that he knew something about.

Although the use of concrete, in one form or another, had been practiced since ancient Egyptian times, it was still thought of merely as a substitute for stone up to the middle of the last century. In 1868, a Frenchman, named Joseph Monier, began placing small iron rods into concrete slabs for added strength and to prevent cracking. This opened a new line of thought for engineers which made concrete adaptable for new uses which were not thought of before. By the turn of the century, the use of steel reinforcement had become an accepted practice (4).

Still many unknowns continued to plague the users of concrete. Bond stresses, diagonal tension, shear, and tensile stresses were little understood. Hooks and stirrups were yet to be invented and monolithic construction was still in its infancy. Simple beams, simple slabs, and simple columns were being connected in much the same manner as steel and wood. Failures were common because proper knowledge of concrete material and skill in its manufacture was not available. The resultant concrete was a bulky, heavy material, lacking the strength and other properties which it should have attained (7).

Even just a few decades ago, concrete was merely a shoveled together mass of cement, sand, stone and water, which in a short time attained a varying degree of hardness and an uncertain strength (7). Quality control on the job was practically non-existent. Standard tests had not yet been adopted, such as the slump test, compression, and freeze-thaw test. Even the best designs of the day were often ruined by some workman who threw an extra bucket of water into the mix. Open stockpiles of sand and gravel were exposed to the rain, yet no adjustment was made for moisture. Mixes were dropped or chuted long distances, causing separation. Vibration was inadequate, resulting in honeycomb. Improper curing led to shrinkage and cracking that eventually led to failure. All these things, and others too, caused concrete to be slow in being accepted as a building material. The uncertainty of the results added nothing to its popularity.

The use of concrete grew slowly until World War I, when steel became a critical, as well as a scarce item (4). Due to the demands of war, designers again turned their attention to concrete. Thus, there came into being many concrete structures to meet the needs of the time. Many of these were built by the uncertain standards of prior practice as mentioned above rather than by design developed through research. Due to lack of steel, or lack of knowledge, or both, some of the builders in that period put up buildings with beams that had no stirrups or diagonal tension bars. Oversized beams were cast on the assumption that the additional concrete would make up for lack of steel. This practice was often unreliable but was used of necessity. Many of the buildings constructed during that period have continued to stand and are still in use today, even though their strength may be questionable.

As the years have gone by research and practice have joined hands in the refinement of concrete. Designers of today can look back at some of these old structures and say "What a shame it couldn't be done over again". However, even though an owner would like to tear down a building and replace it with one of modern design, the expense would often be too great to justify it. Sometimes, an owner would like to add more storage to his old warehouse which has the room but the floors just won't carry the load. Remodeling would be almost as expensive as rebuilding, so the work goes undone and sub-standard buildings continue to be used.

With the improvements of modern design, it would seem as though very few buildings would be in existence that would require additional reinforcement to be added. However, a survey would reveal four classes of concrete structures, the new as well as the old, which are in need of remedial action:

1. Old buildings built by old design standards which resulted in limited load capacity.

2. Buildings which were designed to carry a certain load and the load requirements were increased after the building was built.
3. Buildings which were designed to carry a certain load and failed to come up to specifications, either through weakness of designs or faulty construction.
4. Buildings suffering partial failure due to other causes.

Of the class listed, it would appear that the third category would be the least likely to occur. Yet, even in present day practice, there have been failures reported which according to the best theories of design just couldn't have happened but did (1). Since many structures have been built, following the same design practice as those that failed, the need for preventive measures has become exceedingly clear.

From all the foregoing discussion, there stands out a basic fact, that there are in existence today, many concrete structures which are inadequate in their present state and the cost of replacing or remodeling, in whole or in part, is quite often uneconomical. The question has thus arisen, "Is there an economical method of reinforcing a concrete structure already in place without resorting to major structural changes?"

This then, has generated a new field of research. This study covers only a very small segment of that field and is limited to only one method which has emerged as a partial answer to the problem.

STATEMENT OF OBJECTIVE

Recent studies by national authorities in the concrete field have led to a method of reinforcing existing concrete structures by means of high strength steel bands stretched tightly around beams (1). Design and application of these straps has been developed to the extent that codes have been written to govern their use. Many refinements have been made to the general method so as to gain maximum benefits from the straps (6).

After reading these studies, it has occurred to this writer that there may be many situations confronting an engineer whereby he cannotalways obtain the proper materials or installation tools to produce equally as good results. Immediate action may be required when time does not permit procurement of the most desirable materials or equipment. Examples of this would be:

1. Buildings cracked due to settling where delay in repair might result in collapse.
2. Earthquake damage requiring immediate reinforcing to prevent further cracking.
3. Structures damaged by bombing. (Military necessity may require immediate reoccupancy)
4. Displacement by fire, flood, landslides, snow, hurricanes, or other disaster causes.

In situations of this sort, a field expedient would be required to make temporary repairs that would serve satisfactorily until more permanent repairs could be made at a later date. This means using common, ordinary tools and materials that would be likely to be found anywhere. One thought in mind was the use of ordinary crating materials.

Taking into consideration the emergency conditions under which the field expedient might be used, the objective of this study is therefore limited to finding the answer to two questions:

1. Would ordinary, lightweight strapping (such as might be found in any commercial packing and crating operation) be suitable as equivalent web or stirrup reinforcement for concrete beams?
2. Could it be applied <u>without</u> any refinements and still obtain satisfactory results?

In order to properly evaluate the findings of this study, it will be necessary to compare the results with tests performed by others where all the refinements were employed under near ideal conditions. Therefore, a review of previous studies must be presented as an integral part of this study. The author is not attempting, nor does he expect to obtain results equal to these other tests. However, using their tests as a standard for comparison, he is trying to find out if by using a field expedient method, with ordinary materials, he can obtain an acceptable degree of strength.

REVIEW OF LITERATURE

Events Leading Up To Previous Investigations (1)

In 1954, the Corps of Engineers, U.S. Army, awarded several multi-million dollar contracts to various contractors for the construction of some Special AMC warehouses at seven (7) different Air Bases in the United States. Buildings were of reinforced concrete and practically all were being constructed by the same plans and specifications. Variations were made to suit local conditions but generally speaking, they were all the same type.

On 17 August 1955, one of the completed warehouses at Wilkins Air Force Depot in Shelby, Ohio collapsed with about 4,000 square feet of roofing falling in, following a structural failure in the supporting girders. Cracking had been observed in this building for several months prior to this event but due to the fact it had been completed for over 18 months, the cracks were not considered serious until the major failure occurred. An investigation was immediately made on all contracts. Cracks in some other buildings were discovered at the following locations:

Warner Robins Air Force Depot, Georgia
Gentile Air Force Depot, Ohio
Tinker Air Force Depot, Oklahoma
Kelly Air Force Depot, Texas
Griffiss Air Force Depot, New York
Brookley Air Force Depot, Alabama

Buildings at all these locations were in various stages of completion. However, as a result of the serious failure at Wilkins Air Force Base, a halt was called on all construction on all seven bases. This resulted in a very costly tie-up.

The firm of Ammann and Whitney, Consulting Engineers, of New York City, was employed by the army to investigate these failures and to make recommendations on action to be taken. A summary of their findings is as follows: (1)

Cause of Failure:

1. Failure was not due to a local weakness but due to a general weakness throughout the entire structure. This condition was true in all buildings as evidenced by the widespread pattern of cracking.
2. Failure was due to a combination of diagonal and axial tension.
3. Axial tension was attributed to an over-stressed condition near the points of contraflexure in the top of the beam. It was most evident near the ends of the top steel bars which were placed for negative moment. The general opinion was that the negative steel was not long enough and since the tensile stresses in the concrete exceeded the tensile capacity of the member, failure came as a result.
4. Tensile stresses were induced by wide variations of temperature which failed to be relieved by "frozen" or partially inoperative expansion joints.
5. Roof slab was not cast integrally with the over-head beams. Thus, the roof sections did not act as a single unit but as independent elements which in some cases induced axial tension as a result of uneven expansion in adjacent bays.

This caused a shifting in the moment distribution pattern which in turn introduced some negative moments outside the top steel.

6. Insufficient stirrups were provided for shear and diagonal tension. This was not a case of underdesigning but the stresses were greater than originally calculated.

Check of Materials:

All materials conformed to specifications and in most cases exceeded the minimum requirements.

Construction Methods:

1. Contractors were not found at fault on any of the contracts.
2. Buildings were constructed according to plans and specifications.

Check of Design:

1. No errors were found in calculations.
2. Design procedure and analysis followed standard practice.
3. Plans and specifications conformed to U.S. Building codes and would have been accepted by any city, state, or national codes.
4. No errors could be found in any plans.

As a result of this report, separate investigations were begun by the Portland Cement Association and the American Concrete Institute. The American Society of Civil Engineers reviewed the findings. In addition, many consulting engineers launched independent studies of the data obtained. It was generally agreed by all concerned that the failure was not a result of carelessness in design or construction but due to an inherent weakness in the code themselves.

Since all this has happened, the PCA and ACI, as well as many other national professional organizations, have begun a revision of their codes. This would indicate that there may be some other buildings built under the present code which may also be overstressed and might be inspected for possible strengthening. As a matter of fact, investigations have already been made at four other locations:

1. Baird Bakery, Dallas, Texas
2. Buffalo Flight Hangar, Municipal Airport, Buffalo, New York
3. Marine Corps Warehouses, Albany, Georgia
4. University of Illinois Buildings, Urbana, Illinois

Similar cracking has been observed in these buildings but as yet no action has been taken to remedy them.

Meanwhile, the army, having stopped millions of dollars worth of work, had to find a solution to their problem. On those buildings where the girders had not yet been cast, the plans were revised to incorporate more vertical steel to take care of diagonal tension. However, on the beams that were already up, the problem of adding more steel was not so simple.

It was estimated that to tear down the existing structures and rebuild would cost over $6,000,000. Therefore, the designers began to try to find some other economical method of reinforcing the present structures in place. This would save in time and money as well as utilize what had already been built.

Three methods were initially considered:

1. Use of steel yokes, placed on top and bottom of the beam and connected by vertical bolts on the two beam sides.

2. Bending bar stirrups around the girder with a welded splice at the top plus two right angle bends on each leg. Continuous longitudinal bars to be placed at the four corners inside the stirrups. The girder then to be wrapped with wire fabric and bonded into place with pneumatically applied mortar.
3. Using pretensioned high strength wire strand, straight and/or draped to increase axial compression and thus reduce vertical shear.

On the basis of past experience with these methods, all were rejected. They were time consuming and high in cost. Furthermore, in the case where the roof had already been placed, there was insufficient headroom to allow the application.

The man who finally came up with the answer was Mr. William E. Schaem, a civilian engineer in the Office of the Chief of Engineers, Washington, D.C. He suggested using high strength steel bands, applied externally to the beams, in much the same manner that crates would be banded for shipping. This could be applied without removing the roof, with the minimum of materials, man-power, and equipment. His suggestion was tested in the laboratory with favorable results and was adopted by the army. For this suggestion, Mr. Schaem was given a bonus check for $7,540.00 and was presented to him personally by Secretary of the Army Brucker.

Construction was resumed and repairs were made to the existing structures, using these steel bands, at a cost of only $150,000 as opposed to $6,000,000 replacement costs.

The Portland Cement Association and the Corps of Engineers ran further tests on the "Schaem" method to determine what changes or revisions should be made to present codes. A summary of their findings has been included in pages following.

SUMMARY OF TESTS BY THE PORTLAND CEMENT ASSOCIATION

(Condensed from report in Journal of the American Concrete Institute, Title 53-35, Vol. 28, Jan. 1957) (3)

Test beams were built according to the original plans and specifications for Wilkins Air Force Depot. These beams were duplications of the section that failed, as shown below.

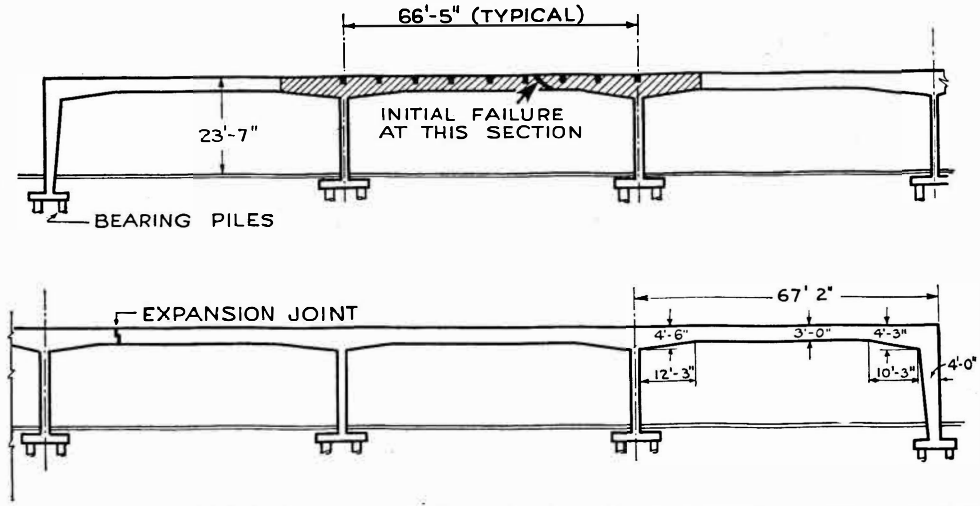


Fig. 1—Typical 400-ft frame at Wilkins depot. Shaded portions represented by laboratory specimens

Figure 1

Beams were cast at one-third scale with details as shown:

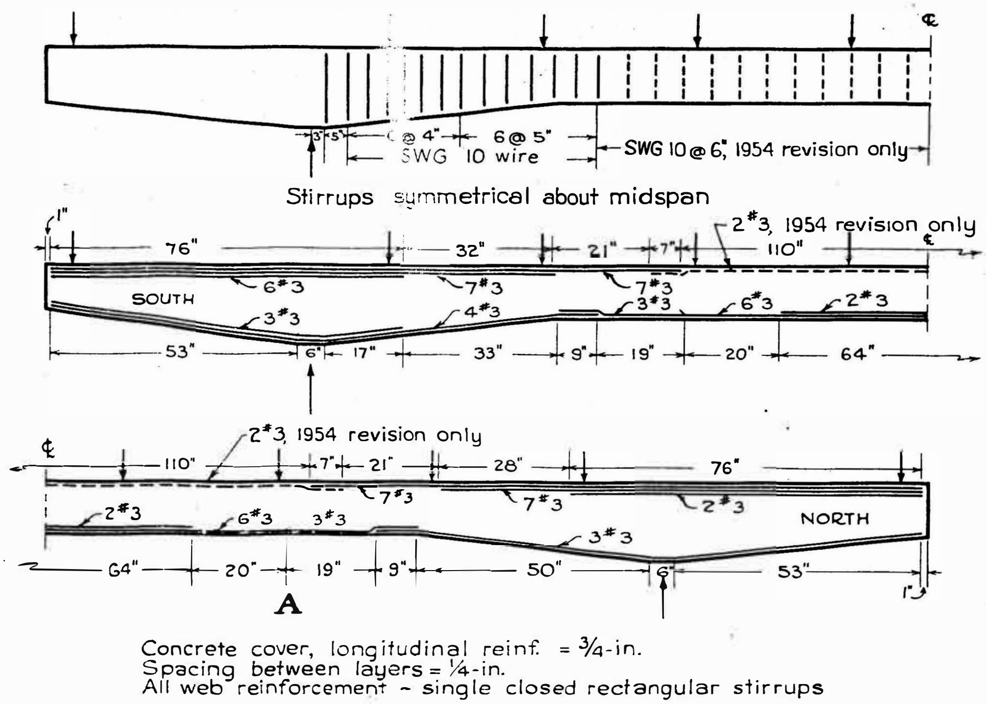


Figure 2

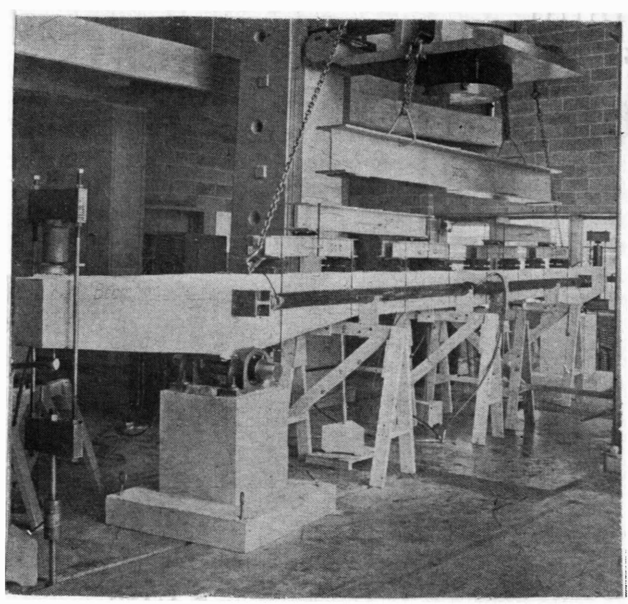


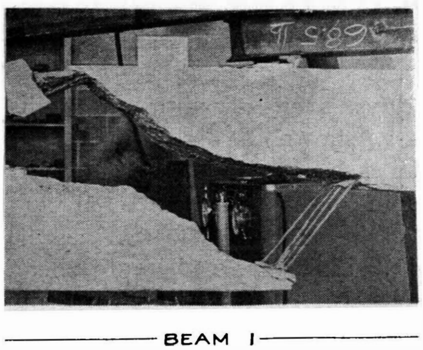
Figure 3

Each beam was then set up for 8 point flexural loading. Although beams in actual structure were continuous, the test beams were not. This was simulated by cantilevering the ends over the supports and tying them down with jacks. This set up may be observed in the picture to the left.

- Test Results -

(Strap size for all beams: 0.75 x .035, A_s = .026)

Beam No. 1: Tested without external strapping to obtain normal failure load.



Ultimate Load: 38.7 kips

Figure 4

Beam No. 2: Strapping was applied as shown below:

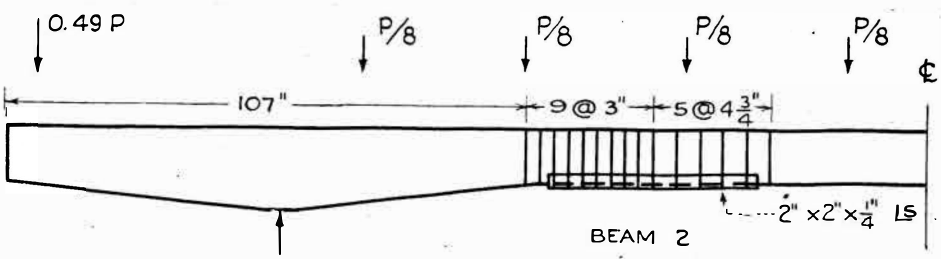
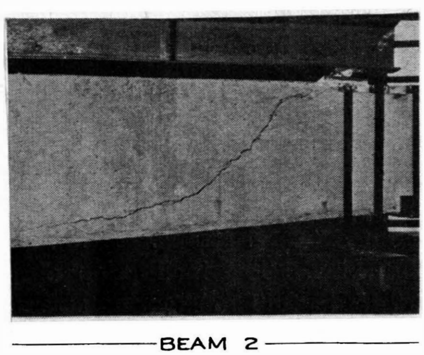


Figure 5



Load was applied to beam and run up to 41.7 kips. When cracking appeared at this point, the load was removed. Pattern showed the failure was due to diagonal tension as seen at left.

Figure 6

Beam No. 2a: Beam No. 2 was repaired by adding some additional strapping over the cracked region and then re-designated as Beam 2a.

Strapping was applied as shown below, in Figure 7:

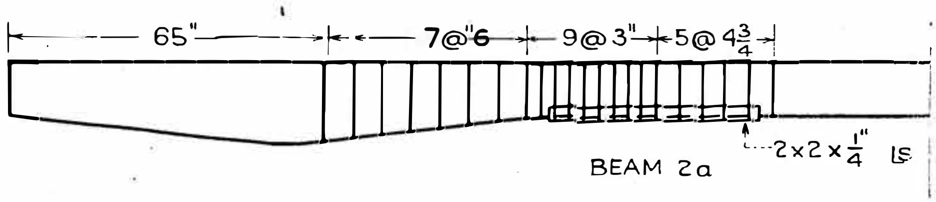


Figure 7

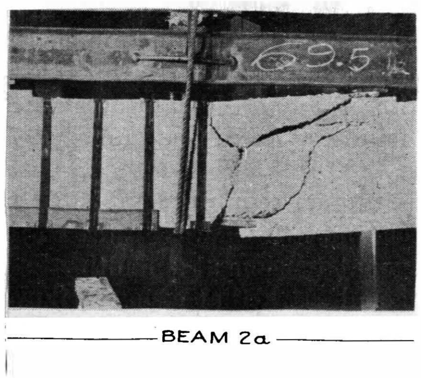


Figure 8

Beam was reloaded and tested to failure.

Ultimate Load: 61.7 kips

Beam No. 3: Strapped as shown in Figure 9:

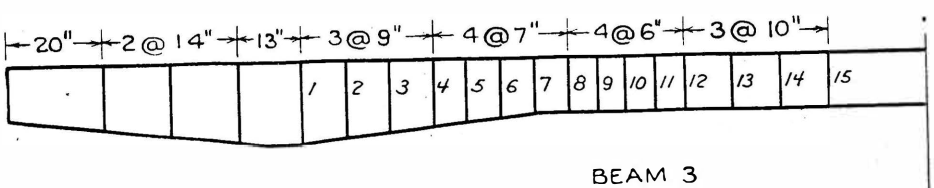
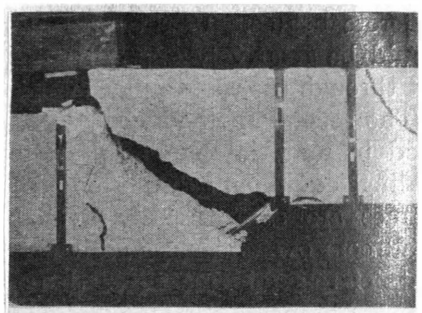


Figure 9

Beam No. 3: (Continued)



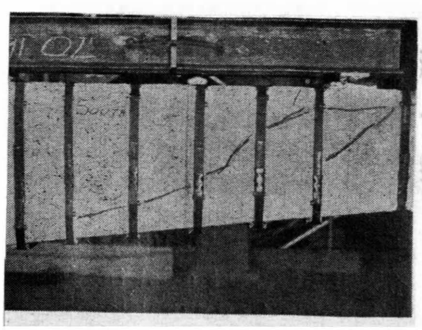
BEAM 3

Failed at 58.7 kips

Figure 10

Beams 4, 5 & 6: (Omitted. Were simply variations of first three tests. Results were substantially the same.)

Beam No. 7:



BEAM 7
(Subsequent Strapping)

First loaded without strapping. Cracking appeared at 39.2 kips. Load was removed and cracked section repaired with straps as shown at left.

Figure 11

Beam No. 7a: Beam 7 was redesignated as 7a and strapping was applied as shown in Figure 12 below:

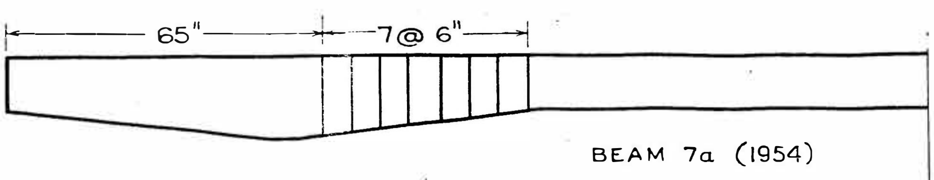
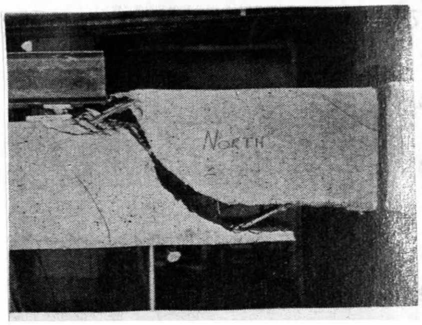


Figure 12

Beam was then reloaded and carried to failure.

Beam 7a: (Continued)



BEAM 7a

Beam failed at 45.7 kips after having been loaded to failure once and subsequently strapped.

Figure 13

Five other beams were tested, using variations of percent of steel. Results were in agreement with the previous tests, so it was not considered necessary to repeat similar findings.

- Summary of Results -
(Failure Loads)

Beam No.	Ultimate Load	Remarks
1	38.7 kips	Without strapping
2	41.7 kips	Partial strapping
2a	61.7 kips	Full strapping
3	58.7 kips	Full strapping
7	39.2 kips	Without strapping
7a	45.7 kips	Straps applied after initial failure and reloaded

Figure 14

SUMMARY OF TESTS BY THE CORPS OF ENGINEERS, U.S. ARMY

(Condensed from report in Journal of American Concrete Institute, Title 53-55, Vol. 28, No. 7, Jan. 1957 (3)

Since the Portland Cement Association had already tested scale models of the actual structures that failed at Wilkins Air Force Depot, the Corps of Engineers decided to run tests on strapping methods only. For this purpose, simple beams were used, since a comparison of the effects of strapping only was the objective rather than the design of the beam itself.

Ten beams were cast for this investigation. The actual strength of the beams was relatively unimportant, since it was a comparison that was desired. The beams were 78" long and had a cross-section of 6 x 12 inches. The average concrete strength was 4,300 psi and all beams were reinforced longitudinally with two #8 bars of 47,000 psi yield point. The straps that were used were High Strength Steel straps with an ultimate breaking strength of 124,000 psi. The cross-sectional area was .026 sq. in. (.75 x .035).

The number and spacing of straps followed the same design criteria as that used for internal stirrups. Beams were placed on simple supports over a 72" span and loaded at the third points.

- Test Results -

Beam No. 1: Tested without strapping to find normal failure load.
Ultimate load: 26.2 kips

Beam No. 2: 28 Straps applied. Loaded to 50 kips. No failure.
Unloaded and 14 straps removed. (Renumbered: 2a)

Beam No. 2a: 14 straps remaining. Loaded to 50 kips again. No failure. Unloaded and 6 more straps removed. (Renumbered: 2b)

Beam No. 2b: 8 straps remaining
Failure occurred at 48.8 kips

The remaining eight beams were used as confirming tests and all results were in complete agreement with the first tests. Thus, the data thereon has been omitted. Tests showed that the ultimate strength of simple beams could be doubled by use of external strapping.

- Field Application -

On the basis of the above investigation, the Corps of Engineers developed a set of specifications to be used in the application of straps in the field. A contract was let to install straps on the AMC warehouses awaiting completion, using the following procedure: (6)

1. All contact surfaces on angles, half-rounds, and corner plates were cleaned carefully with power driven wire brushes or grinding wheels to provide a smooth contact surface free of all scale and projections.
2. Installation of straps on each girder was started at one end of a group of straps between roof slab bearing plates and worked toward the other end so that room was always maintained for operation and removal of the stretcher.

   The temporary column clamps holding the corner protection angles on the haunches were left in place until installation of the permanent straps made their removal necessary.

3. Lubricant was applied to contact surfaces of wedges, corner plates, and half-rounds just prior to installation of straps.

4. The straps were tensioned in accordance with the following procedure:

   a. Stretcher was placed on the bottom of the girder to obtain nearly equal stresses in the strap on the sides of the girder.

   b. Strap was placed in the drum slot of the stretcher through the full length of the slot without projecting through and with sufficient slack left in the strap to provide a minimum of $1\frac{1}{2}$ turns on the drum at a maximum torque of 10 ft.-lb. on the indicator. (The bull winch column clamp for holding corner angles temporarily in place is shown in Figure 15 to the left.

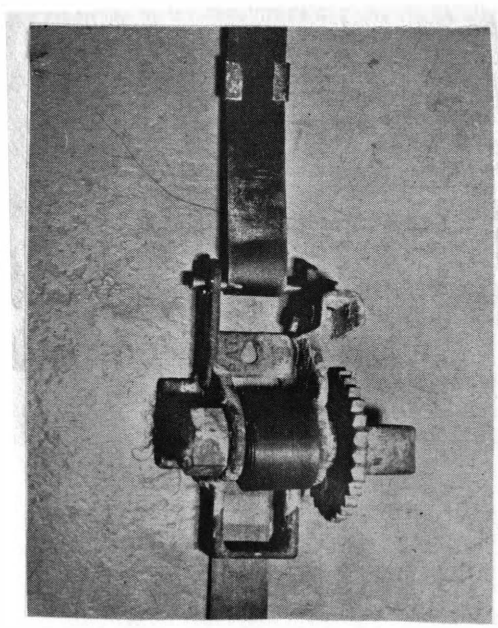
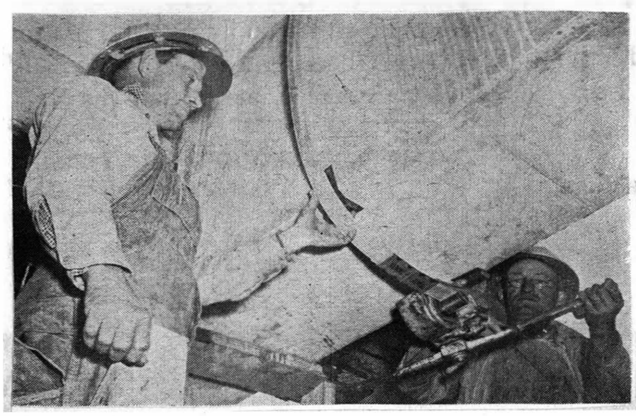


Figure 15

   c. Torque indicator was set at 35 ft.-lb. and strap tightened slowly, the strap being tapped at the corners of the girder during the tightening operation.



Beginning tensioning operation -- note slack in strap.

Figure 16

Tensioning operation. Force being applied on torque indicator.

Figure 17

d. Strap was tightened from the 35 ft.-lb. torque to a final torque of 65 ft.-lb. in increments of 5 ft.-lb. and tapped at the corners of the girders at the end of each increment.

e. Strap was sealed using two seals with two crimps in each seal and the corners again tapped to distribute stress around the girder.

f. The ends of strap and the seals were painted with bitumastic paint immediately after crimping the seals.

When the strapping operation was completed, the Corps of Engineers further recommended that the fireproofing be restored by covering the straps with fireproofing plaster.

The above procedure was followed on all contracts and turned out to be highly successful.

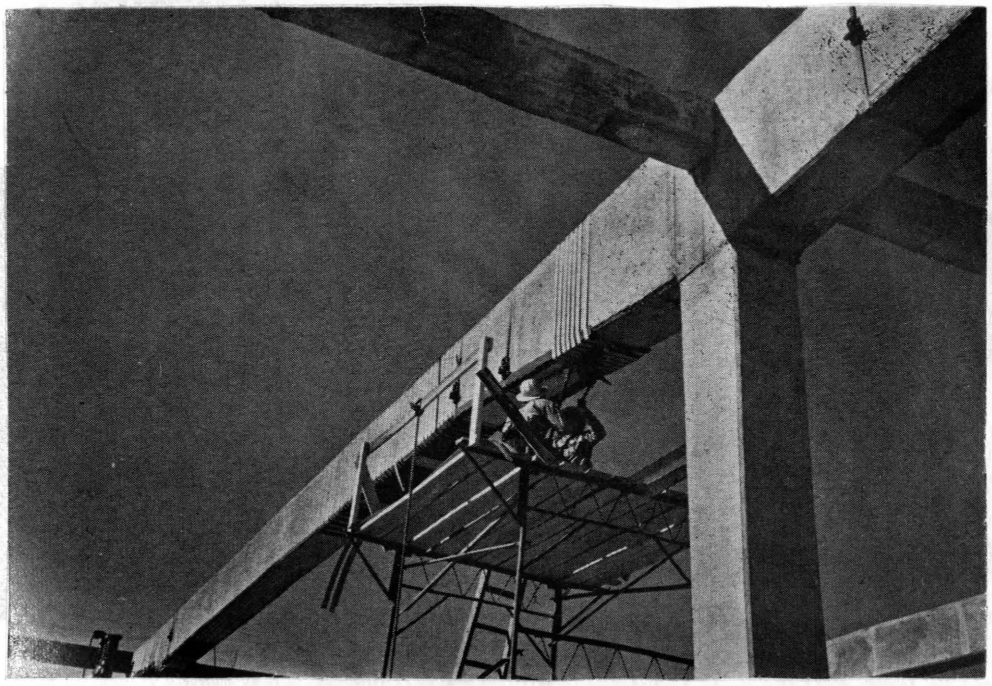


Figure 18

Straps being installed on girder. Note column clamps and straps used for holding angles temporarily in place.

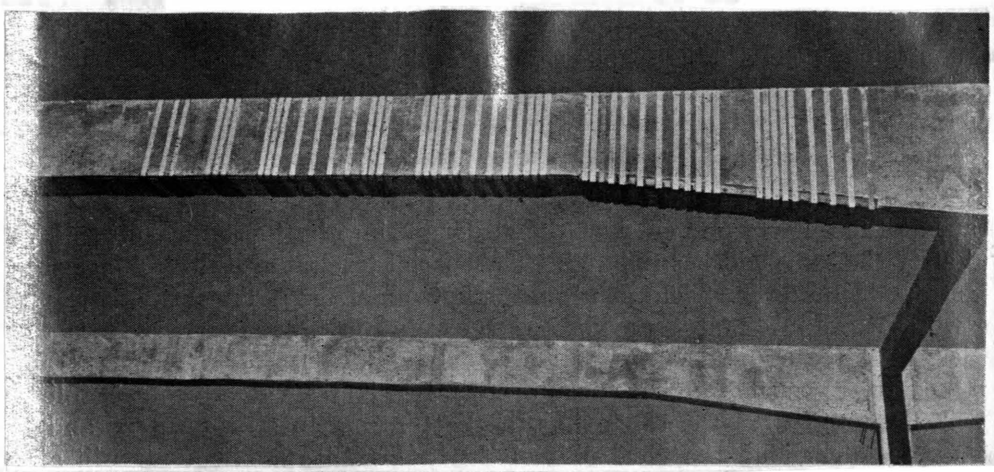


Figure 19

Strapping completed

TESTS AT THE MISSOURI SCHOOL OF MINES

General

A duplication of the tests by the Portland Cement Association and the Corps of Engineers would add nothing except to confirm their results. Since the objective of this test was to use field expedients, without refinements, it was decided to use simple beams, the same size as the ones used in the Corps of Engineers test. This would provide a comparison of strength so an evaluation of the modified method could be made in relation to the standard method.

Thus, four simple beams were cast. Two were tested without straps and two with straps. The results and conclusions are shown later in the report.

Materials

1. Concrete: A trial mix design was used to give a 28 day strength of between 3,000 and 4,000 psi. The actual strength was determined by test cylinders obtained from each mix. The mix was proportioned by weight as follows:

Cement	31.33 #	(1/3 sack)
Water	16.70 #	(2 gals.)
Sand	72.66 #	
Agg	96.66 #	

The coarse aggregate was crushed limestone with a maximum size of 3/4" and a specific gravity of about 2.67.

The water-cement ratio selected was 6 gallons per sack.

Cement was Type I Portland Cement.

Slump averaged 3½" - 4".

Yield was computed at 1.46 cu. ft.

Concrete was mixed in a one cubic foot drum mixer. The dry materials were pre-mixed for about one minute before the water was added. Water was added while the drum was revolving and mixing was continued for two more minutes. Mix was emptied into a large pan and then transferred into the forms by lifting the pan and dumping. A blunt nosed rod was used to consolidate the mix and work it into place between the reinforcing steel.

Four beams were cast. Each beam required three batches. A standard 6" x 12" cylinder was made from every third batch and cured with the beams. All concrete, beams and cylinders were moist cured, in a curing room, wrapped with burlap, at a constant temperature (100^{o}F) and humidity (95%) for 120 days before testing. Initial curing was with wet burlap for three days before removing the forms and placing in the curing room at the end of the curing period, the test cylinders were tested in compression with results as follows:

No.	Load	Area	PSI
1	91,000#	28.27	3,220
2	93,000#	28.27	3,290
3	92,000#	28.27	3,250
4	91,500#	28.27	3,230
		Average	3,248

2. Forms: The forms for the beams were of box construction using 3/4" plywood. The bottom was grooved to receive the sides which were bolted in place to prevent sagging. Two cross-pieces were nailed across the top to prevent any outward movement of the sides. Before being used, the forms received several coatings of oil. The size of the beams were 6" x 12" x 78". Since the beams were to be placed on a 72" span, this allowed a 3" over hang at each end of the beam.

3. Steel: Three #6, structural grade, deformed steel bars were used for bottom reinforcing. They were placed longitudinally, 2" above the bottom. Wire chairs were used to support the rods during the placing of the concrete. Total steel area, each beam = 1.32 sq. in. (f_s = 18,000 psi).

4. Strapping: Strapping material was ordinary 5/8" light-weight strapping. It was the type commonly used in banding small crates for shipping and can be found in general use throughout the country. This particular material was taken from stock in the Packing and Crating Shop at Fort Leonard Wood, Missouri. Since it had no markings to identify its source, no specifications could be obtained on its manufacture. Therefore, tests were run to determine its physical properties. Ten samples were taken at random and tested in tension with the following results:

Ultimate Breaking Strength = 1,246 lbs.

Cross-sectional Area (5/8 x .021) = .01312 sq. in.

Unit Tensile Strength = $\frac{1,246}{.0131}$ = 95,000 psi

STRAP SAMPLE

TESTING OF CYLINDERS

Figure 20

Figure 21

PREPARATION OF BEAMS FOR TESTING

Beams 1 & 2: No straps were applied. These two beams were used to obtain the normal breaking strength, so thus no preparation was needed.

Beams 3 & 4: Straps were applied at 1½" intervals between the end supports and the third points. The center third was left blank since shear in this area was not critical. Computation of number of straps and spacing followed same procedure as for conventional stirrups. (See Table 1)

Since this was to represent a field expedient (in contrast to the Corps of Engineer method) no refinements were used in applying the straps. The corners of the beams were left rough with sharp edges, no grinding or rounding was done, no protecting angles or shims were used, no lubricant was put on the straps, and no strain gages or torque wrenches were used in tightening. The straps were tightened by using an ordinary hand strapping ratchet winch normally used for banding operations.

The winch had a lock gear which held one end of the strap firm. The other end was wrapped around the beam and fed back into the winch under a cog-wheel which advanced the strap each time the wheel was turned. The cog-wheel had a ratchet, which prevented slipping, and could turn in one direction only. The winch was operated by a handle which was worked up and down by hand.

As soon as the strap was tightened to the desired tension, two metal clamps were placed over the over-lapping ends and crimped into place. The winch was then withdrawn. The amount of stress put into the straps was not measured. An attempt to put equal stressing into all the straps was done by trial and error. When the final tightening was begun, it was learned that the handle could not be pressed all the way down without breaking the strap. Through practice, it was learned what position the handle could be placed in to get the maximum tension without breaking the strap and the clamps were applied at that point.

After all straps were in place, the tension was checked by "pinging" or "picking" each strap with a finger. If they were all tightened the same, they would have the same "ring". Any strap which gave off a dull thud, or sounded lower in pitch than the others, was replaced. Thus, all straps were pre-tensioned approximately the same.

Position of straps were as shown below:

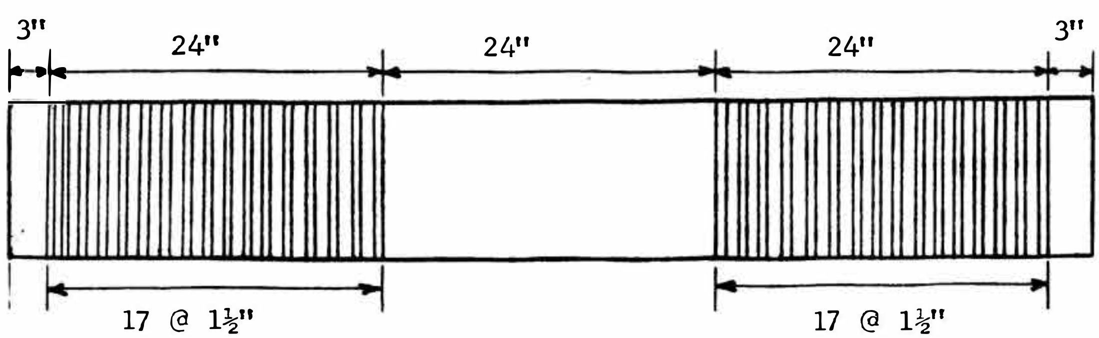


Figure 22

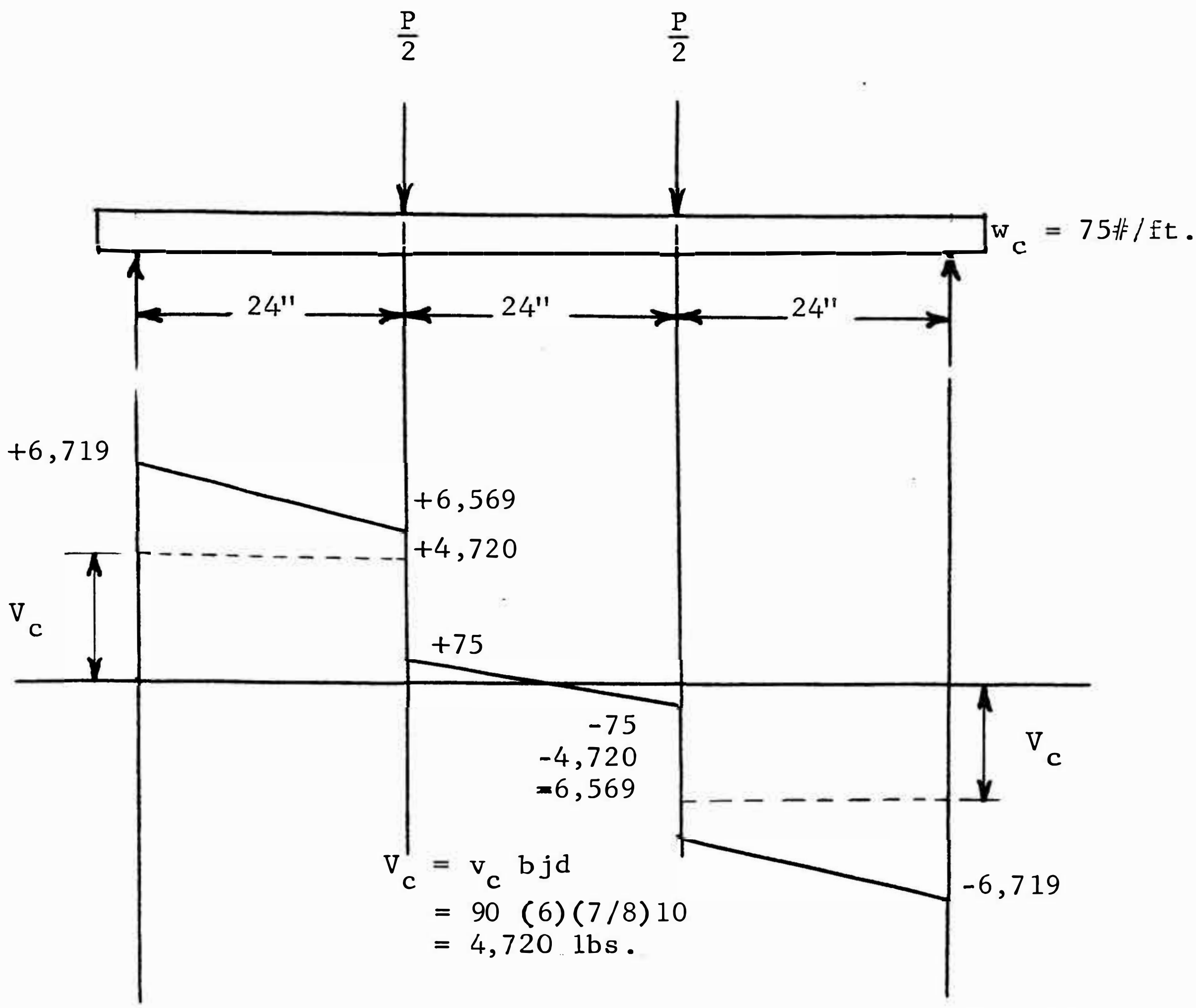
TABLE 1

Computation of Strap Spacing

(Based upon working stress)

$$V_{max} = R_1 = \frac{P}{2} + 244 = \frac{12,950}{2} + 244 = 6,719 \ \#$$

$$v = \frac{V}{bjd} = \frac{6,719}{(6)\,(.875)\,(10)} = 128 \text{ psi} \qquad (j = 7/8 \text{ or } .875)$$



Length requiring strapping = 24" (at each end)

Maximum spacing, ACI Code $= \frac{d}{2} = \frac{10}{2} = 5"$

$(f_v = 95,000)$
$(A_v = .013)$

$$\text{Unit } s = \frac{A_v f_v}{v'b} = \frac{.026\ (30{,}000)}{38\ (6)} = 3.42''$$

This is based on the <u>working</u> stress values <u>only</u>. However, it must be assumed that at ultimate breaking strength, the concrete cannot carry any shear, so thus the straps will have to absorb the full shear values. This condition will come into being after the first cracks appear which means that there can be no transfer of stress vertically between the top of the beam and the bottom except through the straps.

Based upon this assumption, the spacing is recomputed using ultimate strength values, as follows:

$$V_{max.} = \frac{P}{2} + 244 = \frac{30{,}800}{2} + 244 = 15{,}644\#$$

$$v = \frac{V}{bjd} = \frac{15{,}644}{6(.875)10} = 294.4 \text{ psi}$$

$$s = \frac{A_v f_v}{v\ b} = \frac{.026\ (95{,}000)}{294.4\ (6)}$$

$$= 1.39''\ (\text{use } 1\tfrac{1}{2}'')$$

Test Procedure

Third point loading was used for all four beams. Beams were supported 3" from each end, making a clear span of 6' 0" (72 inches). Two steel loading bars were laid across the top on the third points. A 5" I-beam was laid across these loading bars and the main load was applied to the top of the I-beam at the center-point.

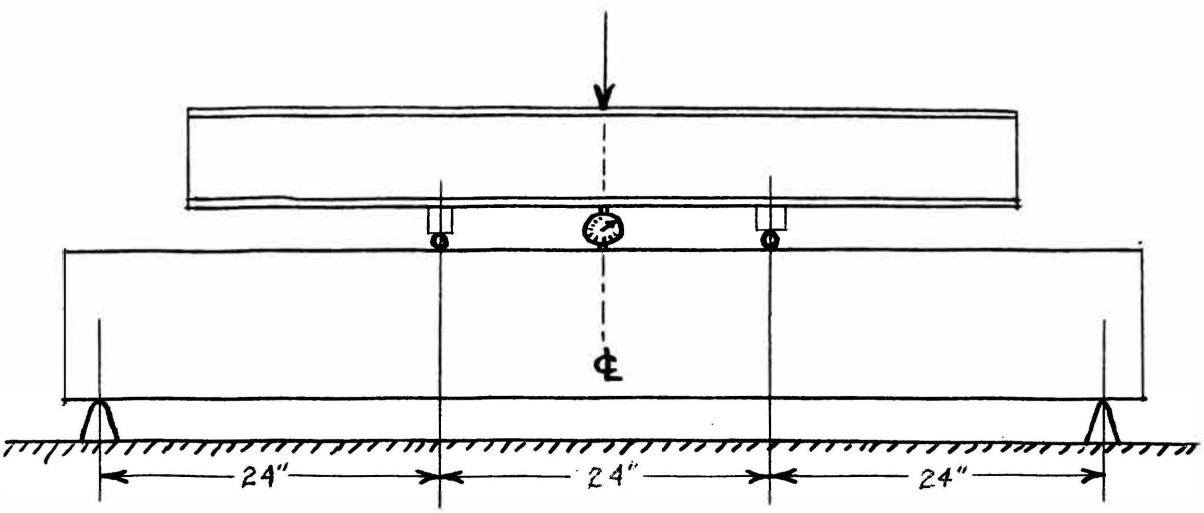


Figure 23

An Ames dial was placed on top of the test beam in the center and supported by a bracket extending out from a stationary part of the testing machine. This dial was calibrated to read deflections to the nearest .001 inch.

The load was applied in increments of 1,000 lbs. and deflection readings were recorded at each increment of load.

Computation of stresses was made before testing to get an approximate idea of where the failure was to be expected. In the case of Beam No. 4, after the initial cracking appeared, the load was applied in 500 lb. increments and plotted accordingly.

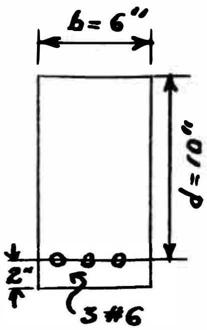
Computations are shown in Tables 2 & 3.

TABLE 2

Computation of Stresses

(Before testing)

a. Initial Data:



f'_c = 3,000 psi (By test)

f_s = 18,000" (Mfg spec)

A_s = 1.32 $in.^2$ (Mfg spec)

n = 10 (ACI Code)

p = .022 (Computed)

k = .463 (Tables)

j = .846 (Tables)

p
$p = \frac{A_s}{bd}$
$= \frac{1.32}{6\ (10)}$
$= .022$

b. Allowable Bending Moments: (Rational Method)

$M_s = A_s\ f_s\ jd = 1.32(18{,}000)\ .846(10) = 201{,}000$ in.-lbs.

$M_c = \frac{1}{2} f_c kjbd^2$

$= .5(1{,}350)\ .463(.846)6(10)^2$

= 159,000 in.-lbs.

f_c
$f_c = \frac{f_s k}{n(1-k)}$
$= \frac{18{,}000(.463)}{10(1-.463)}$
$= 1{,}550$
Use 1,350 since f_c exceeds specs

$M_c < M_s$

$M_{(allowable)}$ = 159,000 in.-lb.

c. <u>Reactions:</u> (Assumption: Wt. of concrete = 150 pcf)

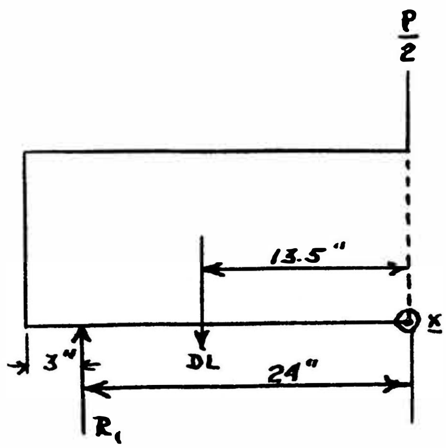
W = Wt. of beam

$$= \frac{6.5 \times 150}{2}$$

$$R_1 = R_2 = \frac{P + W}{2}$$

$$= \frac{P}{2} + 244$$

d. <u>Allowable Working Load:</u> ($M_{(allowable)}$ = 159,000 in.-lb.)



$$DL = W \times \frac{27}{78}$$

$$= 169\#$$

$$\Sigma M \;@\; \underline{x} = R_1(24) - DL\,(13.5)$$

$$159{,}000 = \left[\frac{P}{2} + 244\right](24) - 169(13.5)$$

$$159{,}000 = \frac{24P}{2} + 5{,}860 - 2{,}280$$

$$12P = 155{,}420$$

$$= 12{,}950 \text{ lbs. (13 kips)}$$

TABLE 3

Ultimate Bending Strength (5)

(Without straps)

a. Calculation of p: (From ASTM A15, f_y = 33,000 psi)

ACI allow $p = \dfrac{.4\ f'_c}{f_y}$ p (actual) $= \dfrac{A_s}{bd}$

$= \dfrac{.4(3,000)}{33,000}$ $= \dfrac{1.32}{6(10)}$

$= .0364$ $= .022$

Since, p(allow $>$ p(actual, use: p = .022

b. Ultimate Bending Moment:

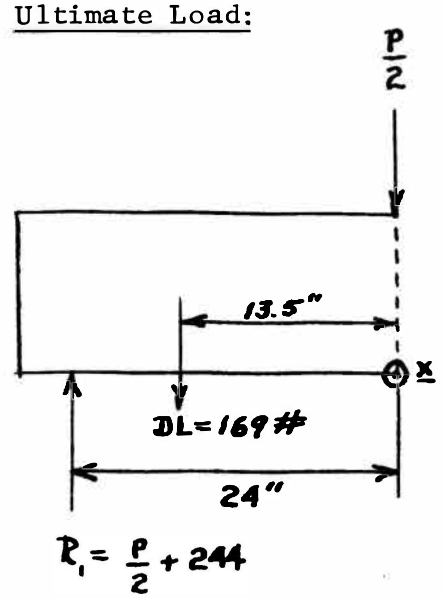
$$M'_s = A_s f_y d \left[1 - \frac{pfy}{2(0.85)f'_c}\right]$$

$$= 1.32(33,000)\,10 \left[1 - \frac{.022(33,000)}{2(.85)3,000}\right]$$

$$= 436,000\ (1 - .1425)$$

$$= 374,000 \text{ in.-lbs.}$$

c. Ultimate Load:



ΣM @ x:

$$M'_s = R_1\ (24) - 169(13.5)$$

$$374,000 = \frac{P}{2} + 244\ 24 - 2,280$$

$$12P = 370,420$$

$$= 30.8 \text{ kips}$$

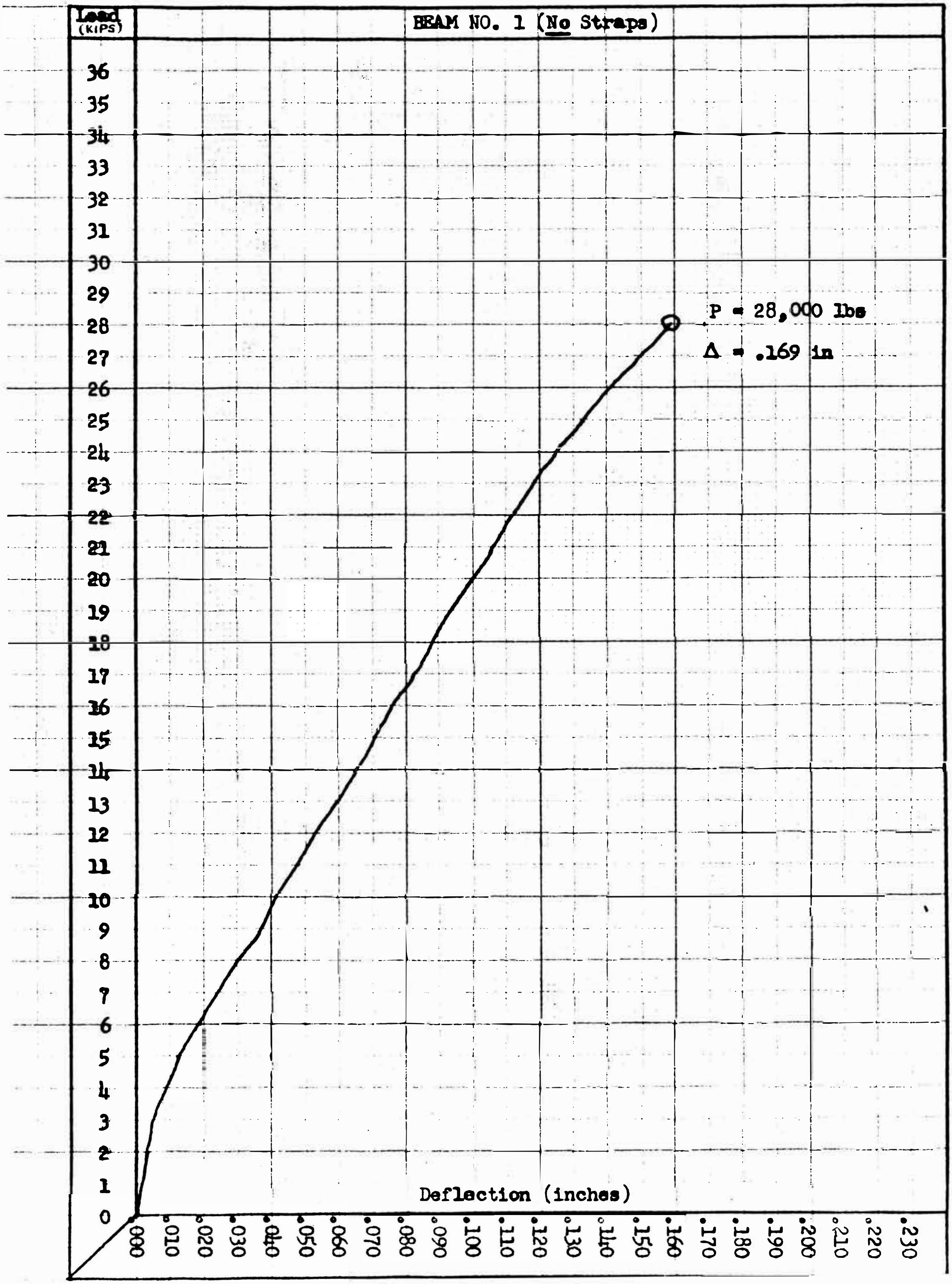
TEST RESULTS

MSM Beam No. 1

Table 4

Load (Kips)	Deflection Dial	Deflection Cumulative	Load (Kips)	Deflection Dial	Deflection Cumulative
0	.802	.000	15	.731	.071
1	.808	.006	16	.726	.076
2	.802	.000	17	.720	.082
3	.797	.005	18	.714	.088
4	.793	.009	19	.708	.094
5	.790	.012	20	.702	.100
6	.783	.019	21	.696	.106
7	.778	.024	22	.690	.112
8	.772	.030	23	.684	.118
9	.765	.037	24	.677	.125
10	.761	.041	25	.669	.133
11	.754	.048	26	.660	.142
12	.749	.053	27	.652	.150
13	.743	.059	28	.633	.169
14	.737	.065	28.3	Failure	

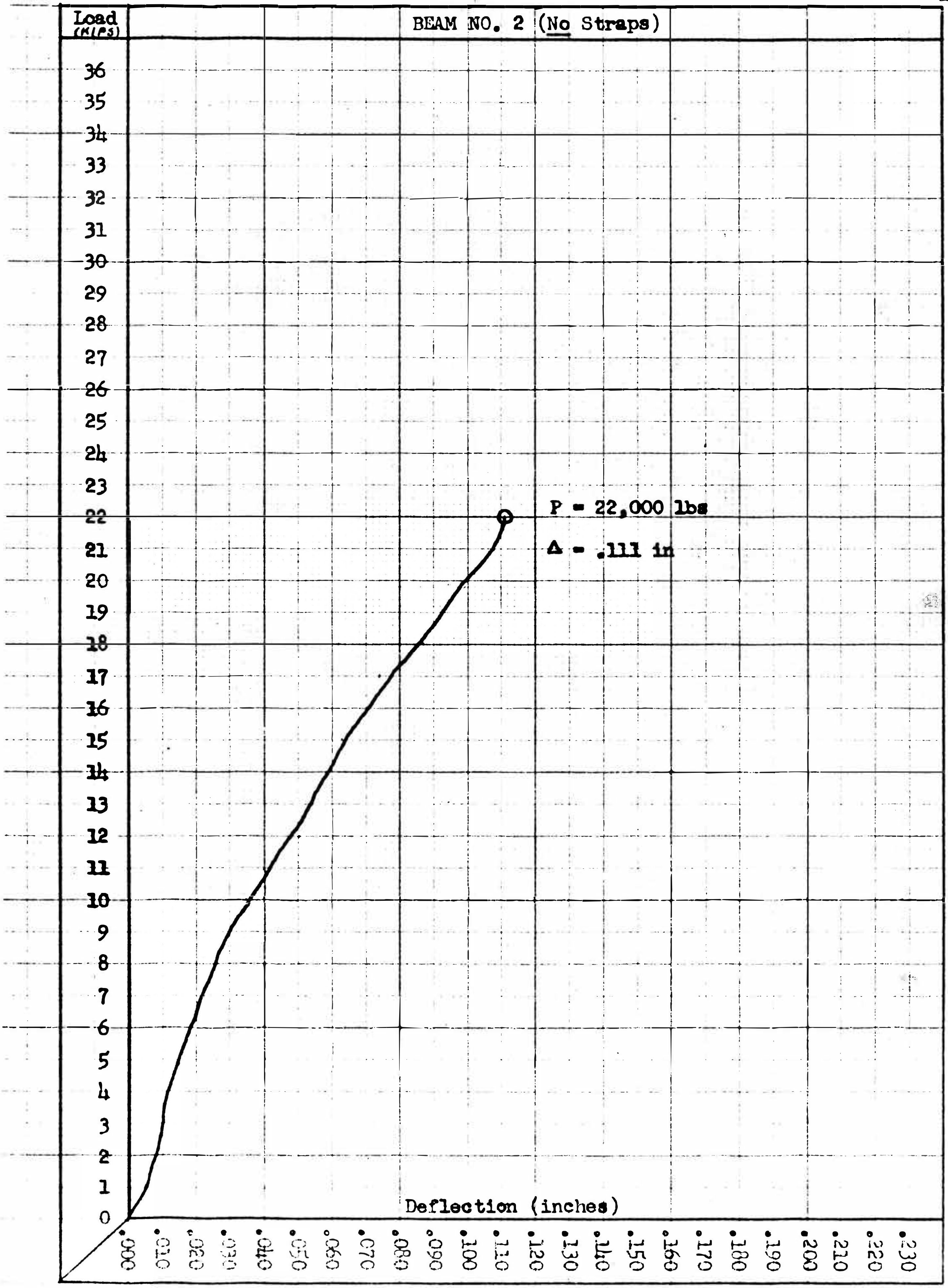
Remarks: Beam No. 1 had some excess concrete due to spreading of the forms during casting. Normal strength is nearer the value of Beam No. 2.


Load (KIPS)
BEAM NO. 1 (No Straps)
36
35
34
33
32
31
30
29
28
27
26
25
24
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
P = 28,000 lbs
Δ = .169 in
Deflection (inches)
.000
.010
.020
.030
.040
.050
.060
.070
.080
.090
.100
.110
.120
.130
.140
.150
.160
.170
.180
.190
.200
.210
.220
.230

MSM Beam No. 2

Table 5

Load (Kips)	Deflection	
	Dial	Cumulative
0	.300	.000
1	.295	.005
2	.292	.008
3	.290	.010
4	.288	.012
5	.285	.015
6	.282	.018
7	.278	.022
8	.274	.026
9	.270	.030
10	.265	.035
11	.260	.040
12	.250	.050
13	.247	.053
14	.241	.059
15	.234	.063
16	.227	.071
17	.224	.074
18	.209	.087
19	.203	.093
20	.197	.099
21	.188	.108
22	.185	.111
Failure		


Load (KIPS)
BEAM NO. 2 (No Straps)
P = 22,000 lbs
Δ = .111 in
Deflection (inches)
36
35
34
33
32
31
30
29
28
27
26
25
24
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
.000
.010
.020
.030
.040
.050
.060
.070
.080
.090
.100
.110
.120
.130
.140
.150
.160
.170
.180
.190
.200
.210
.220
.230

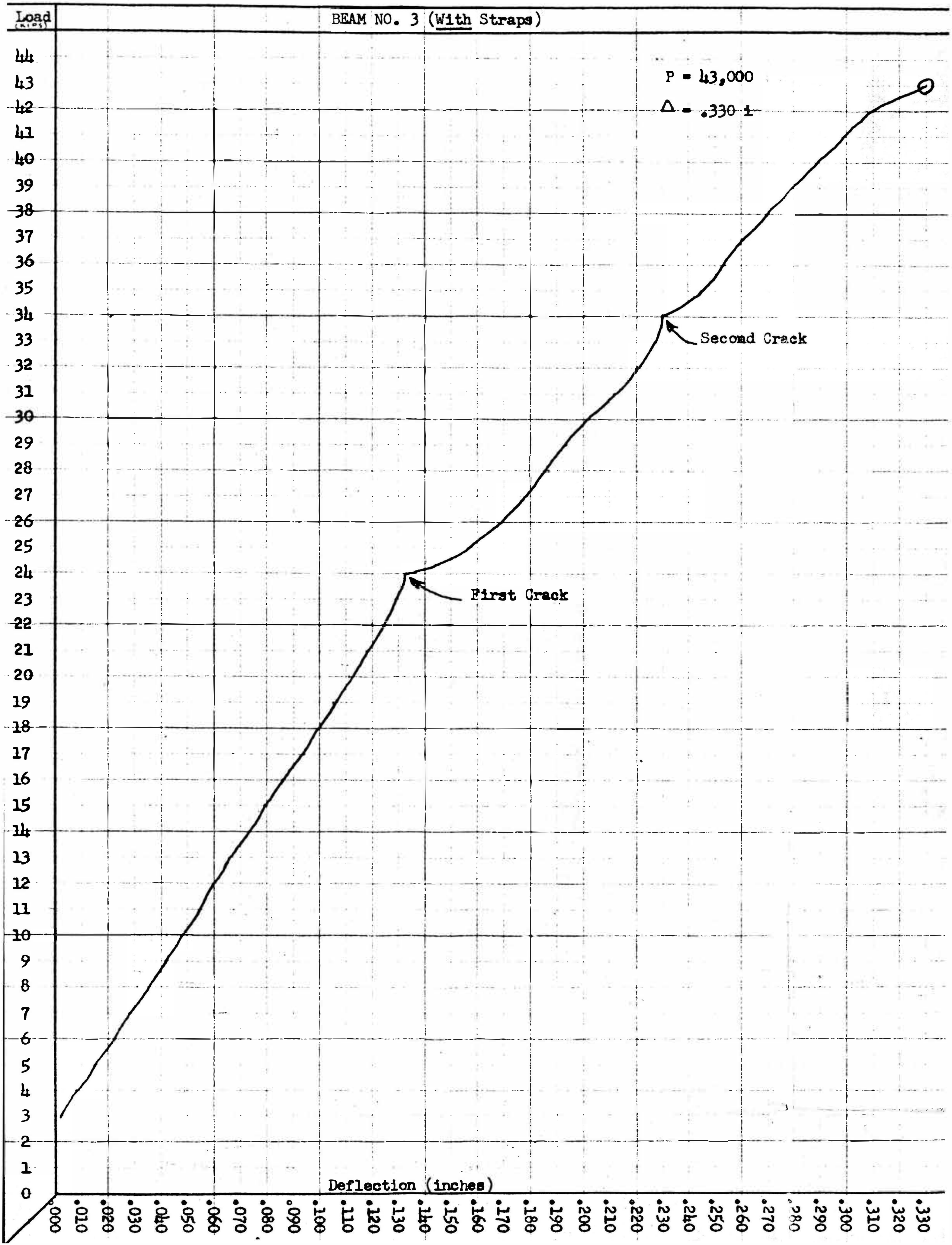
Failure of Beam No. 2

Figure 24

MSM Beam No. 3

Table 6

Load (Kips)	Deflection Dial	Deflection Cumulative	Load	Deflection Dial	Deflection Cumulative
0	.850	.000	22	.729	.125
1	.850	.000	23	.725	.129
2	.850	.000	24	.721	.133
3	.852	.002	25	.697	.157
4	.846	.008	26	.685	.169
5	.839	.015	27	.676	.178
6	.831	.023	28	.667	.187
7	.825	.029	29	.660	.194
8	.823	.031	30	.652	.202
9	.812	.042	31	.644	.212
10	.806	.048	32	.636	.220
11	.799	.055	33	.629	.227
12	.794	.060	34	.626	.230
13	.788	.066	35	.611	.245
14	.780	.074	36	.602	.254
15	.775	.079	37	.595	.261
16	.768	.086	38	.586	.270
17	.760	.094	39	.577	.279
18	.754	.100	40	.567	.289
19	.749	.105	41	.557	.299
20	.743	.111	42	.546	.310
21	.736	.118	43	Failure	


BEAM NO. 3 (With Straps)
Load (KIPS)
P = 43,000
Δ = .330 i
Second Crack
First Crack
Deflection (inches)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36
37
38
39
40
41
42
43
44
.000
.010
.020
.030
.040
.050
.060
.070
.080
.090
.100
.110
.120
.130
.140
.150
.160
.170
.180
.190
.200
.210
.220
.230
.240
.250
.260
.270
.280
.290
.300
.310
.320
.330

Beam No. 3

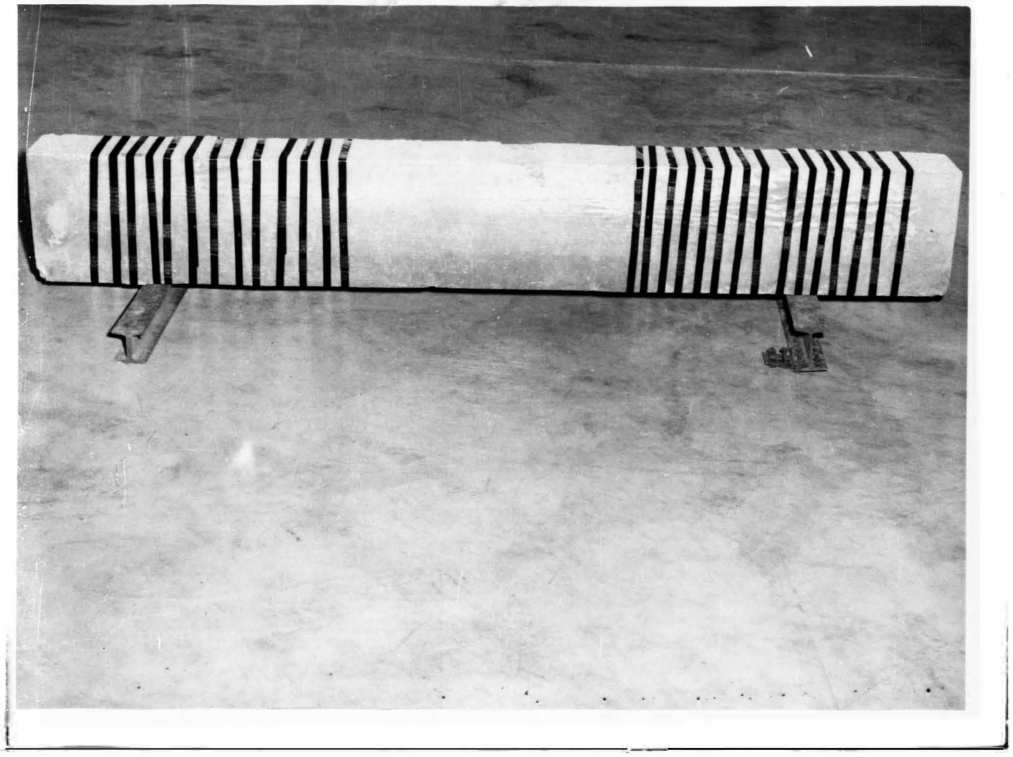


Figure 25 - Beam before testing

Figure 26 - Beam after testing

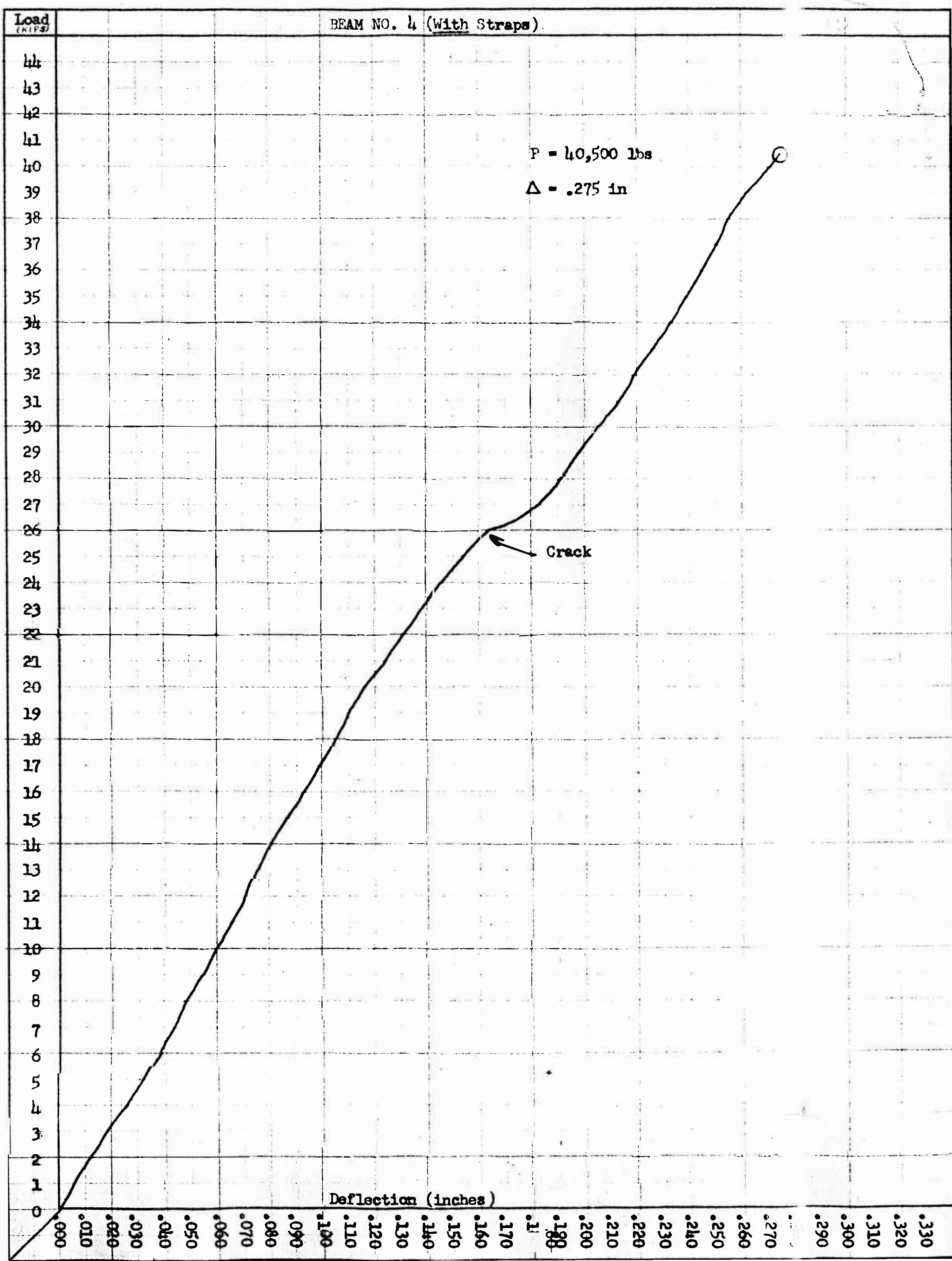
MSM Beam No. 4

Table 7

Load (Kips)	Deflection Dial	Deflection Cumulative	Load (Kips)	Deflection Dial	Deflection Cumulative
0	.703	.000	27.5	.515	.188
1	.697	.006	28.0	.512	.191
2	.690	.013	28.5	.508	.195
3	.684	.019	29.0	.505	.198
4	.677	.026	29.5	.501	.202
5	.672	.031	30.0	.497	.206
6	.665	.038	30.5	.495	.208
7	.659	.044	31.0	.490	.213
8	.655	.048	31.5	.487	.216
9	.648	.055	32.0	.484	.219
10	.643	.060	32.5	.481	.222
11	.637	.066	33.0	.477	.226
12	.632	.071	33.5	.474	.229
13	.628	.075	34.0	.470	.233
14	.622	.081	34.5	.467	.236
15	.616	.087	35.0	.464	.239
16	.610	.093	35.5	.461	.242
17	.604	.099	36.0	.458	.245
18	.598	.105	36.5	.455	.248
19	.593	.110	37.0	.452	.251
20	.587	.116	37.5	.449	.254
21	.579	.124	38.0	.447	.256

Table 7 cont.

22	.572	.131	38.5	.443	.260
23	.565	.138	39.0	.441	.262
24	.558	.145	39.5	.436	.267
25	.549	.154	40.0	.432	.271
26	.539	.164	40.5	Failure	
27	.520	.183			


BEAM NO. 4 (With Straps)
Load (KIPS)
P = 40,500 lbs
Δ = .275 in
Crack
Deflection (inches)
44
43
42
41
40
39
38
37
36
35
34
33
32
31
30
29
28
27
26
25
24
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
.000
.010
.020
.030
.040
.050
.060
.070
.080
.090
.100
.110
.120
.130
.140
.150
.160
.170
.190
.200
.210
.220
.230
.240
.250
.260
.270
.290
.300
.310
.320
.330

Beam No. 4

Figure 27

Note the three different types of strap failures.

1. Corner break
2. Side break
3. Clamp break

The most common type of failure was the corner break.

Figure 28

SUMMARY OF RESULTS

Beam No.	Ultimate Strength	Remarks
1	28.3 kips	No straps
2	22.0 kips	No straps
3	43.0 kips	With straps
4	40.5 kips	With straps

Using Beam No. 2 as normal, the following percent of gain is noted:

Beam No.	% Gain
3	+95%
4	+84%

Comparison to Computed Stresses (From Table 3)

(Loads shown in kips)

Beam No.	Ultimate Stress	
	Load	% Change
Computed	30.8	------
1	28.3	- 8%
2	22.0	-28%
3	43.0	+40%
4	40.5	+31%

COMPARISON TO CORPS OF ENGINEERS TEST

For purpose of discussion, Corps of Engineer Beam 2B will be referred to as Beam "A" and MSM Beam No. 3 will be referred to as Beam "B". A tabulation of their respective values is shown below:

Table 8

Item No.	Item	Beam "A"	Beam "B"	Ratio A/B
1	Concrete Strength	4,300 psi	3,200 psi	1.34
2	A_s	1.57 in^2	1.32 in^2	1.20
3	Strap Strength	124,000 psi	95,000 psi	1.30
4	Strap X-Sect Area	.026 in.^{2}	.013 in.^{2}	2.00
5	Ultimate Load	48.8 kips	43.0 k	------
6	No of End Straps	4	8*	------
7	Kips Per Strap	12.2	5.37	------

*Note: Although Beam "B" had 17 straps on each end, only 8 straps broke. (The remaining straps were ineffective) Thus, the ultimate load was carried by these 8 straps.

It will be noted above that Beam "A" sustained a higher ultimate load than Beam "B" with half the number of straps.

However, without straps, Beam "A" would be expected to carry a heavier load anyhow due to the higher values of items 1 and 2. Thus, the beams, without straps, are considered proportionately equal.

In analyzing the strap strengths, it is noted that a combination of items 3 and 4 give Beam "A" a strength advantage ratio of 2.3 over Beam "B". Hence, if item 7 for Beam "B" was multiplied by this ratio it would put it on common ground with Beam "A" for comparison. So, 2.3 x 5.37 = 11.85 kips per strap. Now to compare:

	Beam "A"	Beam "B"
(Item 7)	12.20	11.85

This shows that Beam "B" is essentially as strong as Beam "A". The difference is less than 10% which shows that if Beam "B" had been strapped with the same strapping material as Beam "A", the results obtained using the field expedient method would have been almost as high as with the standard method.

DISCUSSION OF RESULTS

In comparing the test beams with each other, the strength obtained is almost twice as much with straps as it is without. In comparing the expedient method of applying straps to the standard method, the strengths obtained (by ratio adjustment) is about the same.

However, even though in this case, the expedient method showed equally as good results, it cannot always be relied upon to give values this high. Without measuring the stress in the straps, equal tension in all straps is hard to obtain. The amount of stress is dependent upon the judgment of the individual. Although practice will reduce the differential in strap stresses, it will always be left to chance as to which and how many of the straps will receive the load. Once the normal beam strength has been exceeded, the excess load will be taken up by the straps which are stretched the tightest. These may, or may not, be in the area of maximum diagonal tension and the number of straps receiving the load will be unpredictable. The first straps to break will govern the strength of the beam. The more straps of equal stress that can be brought into play, the better the distribution of the load will be and will result in higher strength. Unequal stresses will cause a premature failure due to a concentration of the load onto fewer straps.

Sharp corners caused straps to break sooner than necessary. Corner failures were the most common observed in this test. However, there were some straps which broke on the sides of the beam as well. Generally speaking though, it is felt that rounded and lubricated corners will help distribute the stress within the straps and thus obtain higher breaking values.

The strapping material itself appeared to be adequate. It only stands to reason however, that by applying wider, thicker, and stronger straps, less number will be needed. Nevertheless, the lightweight strapping used was effective and satisfactory results were obtained.

CONCLUSIONS

1. Ordinary lightweight steel strapping applied externally to simple concrete beams will increase the ultimate strength of the beams from $1\frac{1}{2}$ to 2 times their normal breaking value.

2. Working stress values may be exceeded <u>safely</u> by as much as 50% when straps are used.

3. Strap design follows the same procedure as that presently used for internal stirrups.

4. Straps may be applied with ordinary strapping tools without refinement of beam, measurement of strap stress, or any special procedures, other than common sense care in placing the straps, and still obtain satisfactory results.

5. Strength obtained by expedient method will be as much as 90% of the strength obtained by the standard method. Percent of efficiency will depend solely upon the care used in application.

6. Method is fast, economical, and easy to use. Common labor may be used to apply the straps with the expedient method.

7. Straps must be protected from fire and weather in order to retain its strength.

8. Straps used to repair cracked sections may be relied upon to recover the full original strength of the beam.

9. Use of the straps should <u>not</u> be used as a substitute for stirrups in original design practice. It should be used only to repair or to give added strength to a structure that is already built.

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VITA

John Osborne Buchanan was born on 11 October 1921, in Asheville, North Carolina. He was the son of Mr. and Mrs. C. C. Buchanan. His father was an attorney in Sylva, N.C. and his mother is librarian of Western Carolina College, Cullowhee, N.C.

He attended high school in Cullowhee, N.C. finishing in 1938 and entering college the same year at The Citadel in Charleston, S.C. He attended the Citadel from 1938 to 1943, studying Civil Engineering, but was called to active duty in the army prior to graduation. Since receiving a commission as 2nd Lt, Corps of Engineers in 1943, he has served continuously as an officer to the present date. His present rank is Lt Colonel (Regular Army).

He has spent 18 years in the army, serving overseas during World War II in Europe and the Phillipines, as well as Alaska, Korea, and Japan after World War II. Most of his time has been spent with engineer troop operations, construction, and engineer planning. He has attended various service schools, including the Advanced Engineer School at Fort Belvoir, Virginia.

In 1947, he married Miss Helen Foote from Houston, Texas. They have one son, Charles Osborne Buchanan.

In 1957, he was sent to the Missouri School of Mines to complete his degree in Civil Engineering. The B.S. degree was received in 1958.

Since 1958, he has remained on at MSM as an Associate Professor of Military Science. During this time he was also enrolled as a graduate student. He took the EIT exam in Missouri in

1958 and received his registration as Professional Engineer in the State of Texas in 1960. He is a member of Chi Epsilon, National Civil Engineering Honor Fraternity and the Society of American Military Engineers.